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Performance of Levee Underseepage Controls: A Critical Review

Thomas F. Wolff

September 2002

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September 2002

Performance of Levee Underseepage Controls: A Critical Review

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Final report

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Preface

This report was originally prepared for the Rehabilitation, Evaluation, Maintenance and Repair (REMR) Research Program, sponsored by Headquarters, U.S. Army Corps of Engineers (USACE), during the 1980s. This report was completed in 1986 by Dr. Thomas F. Wolff, Department of Civil and Environmental Engineering, Michigan State University, East Lansing, MI, under the Intergovernmental Personnel Act of 1970. At the time of writing, computer modeling was new to the engineers' desktop; therefore, some of the recommendations for research development are out of date and some are already in practice. The bulk of this report provides assessment on the performance of levees and the USACE underseepage analysis method. A summary of what has been learned through the observation of floods during the 20-year period of 1966 through 1986 is given, and references are appropriate to the year 1986.

Currently, this report is being published under the Innovative Flood Damage Reduction Research Program (IFDR), sponsored by USACE. Historical information on levee performance was determined a critical research need for the Work Unit "Cumulative Effects of Piping under Levees." Therefore, this report is being published under the more recent IFDR Program. Principal Investigator of the piping work unit is Ms. Eileen Glynn, under the direct supervision of Dr. Joseph Koester, Chief, Geotechnical and Earthquake Engineering Branch (GEEB), Geosciences and Structures Division (GSD), Geotechnical and Structures Laboratory (GSL). Dr. Robert Hall, Chief, GSD, provided general supervision, and Dr. David Pittman was Acting Director, GSL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
feet per minute	0.5080	centimeters per second
inches	2.54	centimeters
miles (U.S. statute)	1.609347	kilometers
square inches	0.00064516	square meters
square inches per foot	19.081	square centimeters per meter

1 Introduction

The Federal Government through the U.S. Army Corps of Engineers (USACE) has a large investment in flood-control levees. Where such levees are built on pervious foundations, seepage beneath the levee (underseepage) during floods can produce pressure and flow conditions capable of initiating subsurface erosion leading to levee failure. Two adverse phenomena may occur; one is sand boils which involves the movement of subsurface sand to the surface by flowing water, and the other is heaving which involves the upward movement of a relatively impervious surface layer resulting from subsurface water pressures in excess of its weight. To prevent such occurrences, the USACE has developed a set of procedures to analyze underseepage conditions on a site-specific basis and a set of procedures to design underseepage control measures. For the most part, these procedures were developed in the 1940s and 1950s. Intensive construction of control measures was accomplished in the 1950s and 1960s. Several moderately large and major floods have provided data from which the validity of the procedures and the security of the constructed system can be inferred. Also, since the 1950s many technical advancements have been made in engineering analysis techniques and construction methods that may merit application to underseepage problems.

The Federal Government's levee system will be expected to provide flood protection for many centuries, regardless of its so-called economic life. It will undoubtedly be subjected to floods equaling and exceeding those already experienced. Conditions along the levees are not static but are subject to periodic natural and man-made changes. Such changes may necessitate review, reanalysis, redesign, reconstruction, and/or modification of the system.

Managers responsible for the rehabilitation, evaluation, maintenance, and/or repair of levees subject to underseepage face the following questions:

- a.* Are the levee systems safe against underseepage failure during flood?
- b.* If not, are the methods used to analyze (evaluation) and to design controls (rehabilitation) appropriate and accurate?
- c.* Do piezometric data obtained during floods provide reliable information applicable to the previous two questions? If so, how should it be interpreted?

- d.* If controls are necessary, does modern technology offer better and/or more cost-effective designs than those used in the past?
- e.* If adverse underseepage conditions occur despite all the above, what are the best methods to provide expedient controls?

In response to such concerns, several researchers have prepared voluminous evaluations of the performance of particular levees in particular floods. This report draws on those previous assessments to summarize in one source what has been learned from observations during floods up to 1986. Using that knowledge, the analysis procedures and the performance evaluation procedures are reviewed to identify possible areas of improvement.

2 Historical Perspective

In response to underseepage problems during the 1937 flood, the Mississippi River Commission (MRC), with the approval of the USACE, in 1940 initiated an investigation of the causes of and methods for controlling underseepage and sand boils along Lower Mississippi River Levees. Much of the work was performed by the U.S. Army Engineer Research and Development Center (ERDC)/U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, and involved both analytical and model studies. The work led to an analysis procedure that predicts the quantity of underseepage, the uplift pressure on the base of the top blanket, and the gradient through the top blanket. Where calculated gradients are excessive, controls are provided. Underseepage control measures traditionally employed have been seepage berms and pressure relief wells. To provide hard data regarding underseepage safety and performance of installed control measures, wellpoint piezometers have been installed at numerous locations along the levees. A number of floods have occurred since piezometers and underseepage controls have been installed. In particular, the Mississippi River floods of 1961, 1969, 1973, 1974, 1979, 1982, and 1983 have generated considerable data on levee performance.

Design procedures for berms and wells evolved from work by ERDC, USACE, the U.S. Army Engineer Districts (USAED), St. Louis and Vicksburg, the Lower Mississippi Valley Division (LMVD), and the U.S. Army Engineer Division, Missouri River. Before discussing performance, the evolution of the present analysis and design procedures is briefly summarized in Chapters 3 and 4.

3 Levee Underseepage Analysis Procedures

Bennett (1946) published solutions for steady-state seepage through a two-layer system composed of a semipervious top blanket overlying a pervious substratum. Flow was assumed vertical in the top blanket and horizontal in the substratum. Bennett stated that the substratum must be at least 10 times as permeable as the top blanket for these assumptions to be reasonable. Although Bennett's solution dealt with downward seepage through blankets upstream of dams, it was equally applicable to upward seepage through the top blanket river-side of levees. All later analysis and design equations are based on extensions of Bennett's blanket formulas and make the same assumptions.

Technical Memorandum 3-424 (WES 1956b), documents the analysis of underseepage and design of controls for the Lower Mississippi Valley levees. The focus of the analysis procedure is the prediction of the residual head, h_o , at the levee toe. Dividing the residual head by the thickness of the top blanket, z , yields an exit gradient. Calculating the residual head and the exit gradient requires assigning (estimating or assuming) values for the gross head on the levee, the levee geometry, and the thicknesses and permeabilities of the substratum and the top stratum. If the calculated exit gradient exceeds an allowable value (typically taken as 0.85),¹ underseepage control measures are designed. The analysis procedures extend Bennett's work to include:

- a. Transformation of a layered top blanket of thickness, z , with vertical permeabilities, k_1, k_2, \dots , to an equivalent uniform top blanket, z , with an equivalent vertical permeability, k_b .
- b. Calculation of an equivalent horizontal foundation permeability, k_f , for a stratified foundation.
- c. Calculation of the distance to effective source of seepage entrance, s , for the special cases of a riverside top blanket of infinite length, a riverside top blanket extending to a river at a finite distance, a riverside blanket extending to a block at a finite distance, and seepage entrance through a

¹ Some Districts have lowered the critical exit gradient to 0.5 since the great flood of 1993.

riverside borrow pit of finite width. Similar cases are treated for the calculation of the distance to the effective seepage exit, x_3 .

A significant aspect of the analysis is the selection of a value for the top blanket permeability, k_b . Although laboratory values for clay are typically in the range 10^{-7} to 10^{-9} cm/sec, values on the order of 10^{-3} to 10^{-5} cm/sec must be used in the analysis to provide reasonable results. This is attributed to field permeability being controlled by defects in the top blanket (cracks, root holes, fenceposts, etc.) rather than properties of the intact soil. Designers originally assigned blanket permeability values from tables relating k_b to general material types and ranges of thicknesses (WES 1956a, 1956b). Later the LMVD (USAEDLMV 1976) published curves giving k_b as a function of material type and blanket thickness.

Technical Memorandum 3-424 (WES 1956b) also provided a detailed discussion of the surficial floodplain geology from a three-dimensional (3-D) perspective and its relationship to underseepage and the occurrence of boils. Nevertheless, the recommended mathematical analysis procedures required a two-dimensional (2-D) idealization of conditions with horizontal soil layers of uniform thickness.

Analysis procedures in TM 3-424 (WES 1956b) were summarized in the professional literature by Turnbull and Mansur (1961a). Similar analyses and designs were performed for levees in the USAED, St. Louis, and were documented in TM 3-430 (WES 1956a) and by Mansur and Kaufmann (1957). The TM 3-424 analysis procedures remain intact in the 1978 Engineer Manual 1110-2-1913, "Design and Construction of Levees" (Headquarters, Department of the Army (HQDOA) 1978).

4 Design Methods

Relief Wells

Muskat (1937) published a solution for the potential head along an infinite line of equally spaced fully penetrating wells in a confined aquifer parallel to a line source of seepage. This solution provided the initial approach to well design. To provide compatibility with conditions along levees and construction practices, Muskat's solution was subsequently modified to account for the effects of leakage into and out of the top blanket, the effects of partial penetration, and the effects of a finite-length well line.

Middlebrooks and Jervis (1947) summarized the then-current Corps' design procedures which adjusted Muskat's method to account for partially penetrating wells based on hydraulic model tests by ERDC/WES and electrical-analogy model tests by the USAED, Vicksburg (Jervis 1939). The hydraulic model test results were later published by ERDC/WES (WES 1949) and by Turnbull and Mansur (1954).

Barron (1948) published a solution for the discharge and pressures associated with an infinite line of fully penetrating wells where leakage occurs through the top blanket. As this procedure predicted lower well flows and lower gradients than procedures based on impervious blankets, it allowed greater spacings and more economical designs.

In 1955, the Headquarters, U.S. Army Corps of Engineers, published Civil Works Engineer Bulletin 55-11, which updated design guidance for well design based on the results of more electrical-analogy model studies. The procedure accounted for partially penetrating wells and a leaking top stratum. Solutions were provided for the average and maximum head in the plane of wells as a function of the head on the levee, thickness and permeability of the two idealized foundation layers, well penetration, well spacing, and well diameter. The procedures in TM 3-424 (WES 1956b) are those of Bulletin 55-11, but TM 3-424 provides further detail as to incorporating hydraulic head loss in the well into the analysis. The analysis requires an iterative solution as the head loss in the well, the head between wells, and the well flow are interrelated variables.

In 1963, Engineer Manual 1110-2-1905 (HQDOA 1963) provided extensive tables for design of finite lines of relief wells. The tables were based on

additional electrical model studies. To this writer's knowledge, however, these tables have seldom been used in design practice.

Seepage Berms

TM 3-424 (WES 1956b) provided solutions for design of impervious, semipervious, and pervious seepage berms. Most, if not all, subsequent berm designs have been for semipervious berms (berm permeability equal to the blanket permeability). LMVD provided supplemental design criteria to be used with the solutions by letter to its Districts in 1962 (USAEDLMV 1962). Design curves incorporating the criteria were published by LMVD in 1976 (USAEDLMV 1976).

Barron (1980) published detailed derivations of design equations for impervious, semipervious, and pervious berms including special cases of constant and variable safety factors. Barron (1984) later corrected the 1980 work and supplemented it with analysis procedures for short berms where boiling is allowed at some distance from the levee toe. In his conclusions, Barron took note of the deterministic nature of his solutions and their sensitivity to variations in the variables; consequently, he suggested that "a probabilistic approach be used in design."

5 Levee Performance During Floods

Lower Mississippi River

TM 3-424 (WES 1956b) and Turnbull and Mansur (1961a) reported the analysis of piezometer data obtained at 15 piezometer sites during the 1950 high water and selected sites at other times. Conclusions pertinent to this study include the following:

- a. *Sand boil occurrence.* The locations of sandboils were highly correlated with local geologic conditions. In point bar areas, most sand boils occurred in ridges adjacent to swales. Sand boils also tended to occur between levees and parallel clay-filled plugs and in landside ditches.
- b. *Sand boil gradients.* Where sand boils occurred, measured gradients were in the range 0.5 to 0.8, often about 0.65, and generally lower than the 0.85 value used in the analysis procedure. Two influencing factors were suggested: old boils may be reactivated at relatively low pressures, and the pressure relief resulting from the boil may lower piezometer readings in the area.
- c. *Entrance and exit distance.* Both the entrance (s) and exit (x_3) distances varied with river stage. In certain cases, a reduction in the entrance distance with river stage was attributed to scour in riverside borrow pits. It was observed that calculated entrance and exit distances were quite variable, and that a 0.015-m (0.05-ft) reading error in each of two piezometers could result in substantial error in calculating these distances.
- d. *Permeability ratios.* Ratios of the substratum horizontal permeability to the landside top stratum vertical permeability, backfigured from the entrance and exit distances, were typically in the range 100 to 2,000.
- e. *Permeability.* Apparent top blanket permeability decreased as top blanket thickness increased as a result of the decreased effect of defects, such as root holes and cracks. Also, the permeability of the landside blanket was 2 to 10 times that of the riverside blanket, apparently

because of downward flow sealing defects and upward flow opening defects.

Appendix E of TM 3-424 (WES 1956b) reported the analysis of the same sites for the 1961 high water. It was found that residual heads at the levee toe were slightly higher than in 1950. Surprisingly, perhaps, no indications of excessive seepage or sand boils were reported in 1961, which is in considerable contrast to the case in 1950. Effective seepage entrance distances (s) were in the same order of magnitude as in 1950, although large increases and decreases were observed in certain instances. Effective seepage exit distances (x_3) were highly variable, with magnitudes one to two times those measured in 1950. The discrepancies in the entrance and exit distances were variously attributed to unsaturated aquifer conditions, riverside scour, faulty piezometers, and unreliable measurements of the tailwater elevation.

USAE District, St. Louis

Wolff (1974) and the USAED, Saint Louis (1976) reviewed the performance of the Alton-to-Gale (Illinois) levee system along the middle Mississippi River during the record flood of 1973. The review was based on approximately 20,000 piezometer readings obtained from approximately 1,000 piezometers along 384 km (240 miles) of levee. Readings from a significant percentage of the piezometers were extrapolated to design flood stages. To minimize unsteady flow effects, only data obtained during the rising side of the river hydrograph were extrapolated. The 1976 report concluded that the analysis and design procedures generally produced a reliable levee, but identified several sets of special circumstances where existing procedures appear deficient:

- a. *Characterization by two soil layers.* Of the reaches found to be apparently still critical with respect to underseepage, many have a thick (6- to 15-m (20- to 50-ft)) layer of sandy silt or silty sand beneath the top blanket and above more pervious sands. In the present analysis and design procedure, this "intermediate" stratum must be mathematically transformed and combined with either the top blanket or substratum. When wells were designed and installed, the intermediate stratum was blanked off as the materials were too fine for the standard filter and screen. During floods, such wells may flow profusely yet piezometers at the base of the top blanket indicate excessive residual heads. This phenomenon occurs because the horizontal permeability of the intermediate stratum is greater than the vertical permeability of the substratum, causing seepage in the intermediate stratum to be more readily conducted landward than toward the well screen (Figure 1). Similar foundation conditions had been tested in the ERDC/WES hydraulic model B (WES 1949), but the wells had not been blanked off.

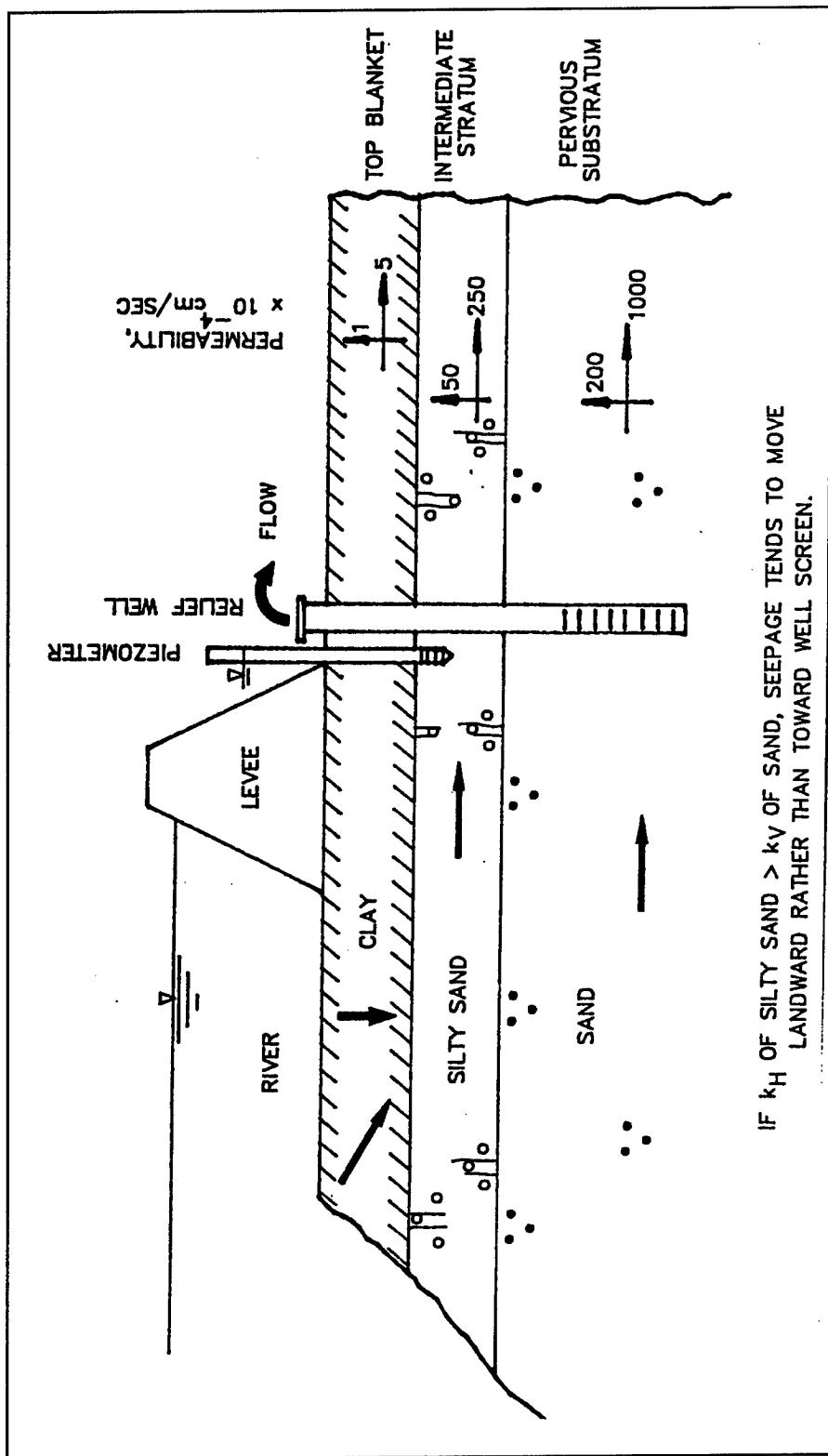


Figure 1. Foundation profile with intermediate stratum

- b. *Corners.* Where a levee bends or turns a corner (frequently encountered where a riverfront levee meets a flank levee), the landside toe is subject to seepage from two directions and the measured residual heads may be significantly higher than those predicted from the 2-D analysis.
- c. *Back levees and flank levees.* Where levees are built to provide protection from small creeks and streams traversing the main river valley that are not efficiently connected to the pervious substratum, piezometric levels may reflect slowly rising regional groundwater levels rather than being a function of the variables involved in underseepage analysis.
- d. *Entrance and exit distances.* Entrance and exit distances calculated at piezometer ranges were frequently found to be shorter than assumed for the original design. Where values of 182 to 305 m (600 to 1,000 ft) were assumed in design, measured values were often 122 m (400 ft) or less.
- e. *Permeability ratio.* The ratios k_f/k_b were smaller than assumed for design (400 to 2,000) (WES 1956a) but were reasonably consistent with later design guidance (100 to 800 in Rock Island) (USAED, Rock Island 1960; $k_b = f(z)$, USAEDLMV 1976). It was also noted that data from piezometer ranges in reaches with flowing wells cannot be used to check design assumptions for s and x_3 because of the significant nonlinear effect of the well drawdown on the piezometric surface.

USAED District, Rock Island

Underseepage conditions in the USAED, Rock Island, were assessed by Cunny (1980) and Bawmann (1983) and their assessments were reviewed by Daniel (1985).

Cunny (1980) prepared a comprehensive report involving 29 piezometer ranges and data from as many as nine high water periods; however, for a number of reasons, the amount of reliable data is much smaller than the above figures would suggest. Salient points and conclusions from Cunny's report include the following:

- a. *Permeability ratios.* No trend between the ratio k_f/k_{bl} and the blanket thickness z could be identified as was found for the Lower Mississippi Valley levees. However, Cunny (1980) recommended that the value of the permeability ratio be taken as 100 on the landside and 200 on the riverside. These values are lower than previous Rock Island criteria (USAED, Rock Island 1960) and significantly lower than Lower Mississippi Valley criteria.
- b. *Residual heads.* Residual piezometric heads at the levee toe were only slightly smaller than calculated using old permeability ratio criteria because of compensating riverside and landside effects. This is further discussed in Table 1.

Table 1 Validity of Assumptions in Underseepage Analysis and Design	
Assumptions	Cases Where Inappropriate
Steady-state flow	Rising and falling river, areas with observed timelag in piezometric response
Two-dimensional flow	Corners or bends in levee
	Seepage concentrations adjacent to clay-plugs and clay filled channels
Two-layer foundation	Where an intermediate stratum (usually silty sand) is present
Vertical flow through top stratum	May be inappropriate where an intermediate stratum is modeled as part of top stratum
Horizontal flow through substratum	May be inappropriate where an intermediate stratum is modeled as part of substratum
Continuous and uniform top blanket	Riverside borrow pits
	Landside ditches
	Ridge and swale topography
	Clay-filled channels parallel to levee

- c. *Berm design.* Required seepage berm widths based on observed data and conditions are much smaller than those calculated from prevailing criteria. Berm width formulas based on maintaining a factor of safety against uplift may not identify where berms are or are not needed. Berms may not prevent boils, but may only move them away from the levee. It appears that berms (or wells) may not be needed at all where pressures can be uniformly and harmlessly dissipated. Sizing berms using a creep ratio approach may be somewhat better than the uplift approach, but further research is required relative to a rational berm design procedure.

Daniel's (1985) review of the Cunney's report and other Rock Island data yielded the following observations and conclusions:

- a. The correlation between measured gradients and the occurrence of sand-boils is weak. Although the analysis suggests initiation of boiling at gradients about 0.85, boils were observed at gradients of 0.54 to 1.02 (avg. 0.68).
- b. Calculation of gradients is sensitive to the top stratum thickness, z , an uncertain quantity.
- c. There is an inverse correlation between blanket thickness and boil occurrence.
- d. Hydraulic conductivity is hard to quantify; values given in Corps criteria are arbitrary.
- e. The hydraulic head is not constant along vertical planes as assumed in analysis.
- f. The effective exit distance, x_3 , is a function of several uncertain parameters and therefore is extremely uncertain.

- g. The effective entrance distance landward of the levee toe, x_1 , apparently varies with river stage in violation of the design assumptions that it is constant.
- h. The critical gradient is based on a homogeneous top blanket with no cohesion and flexural strength.

Daniel's (1985) recommendations include daily reading of piezometers during high water to obtain a better database, further study of the relationship of high water to slope stability, and development of a relatively sophisticated computer program to replace the existing method of analysis.

6 Discussion Regarding Measured Performance

Harr (1977) presented the concept of a design chain (Figure 2). Measurements and performance observations made during floods provide the experience component (fourth link) from which the strength of the preceding links can be gauged. Where performance differs from that predicted, a weakness or anomaly in the chain is indicated. A more detailed analysis chain specific to underseepage analysis is shown in Figure 3. Working backwards through the chain, the following paragraphs discuss the apparently adequate and inadequate aspects of the existing analysis procedures based on measurements and observations made in the Lower Mississippi Valley and the St. Louis and Rock Island Districts.

Occurrence of Sand Boils

Sand boils occur at less-than-predicted gradients. This was noted as early as 1952 (WES 1952) and is well documented in Figure 47 of TM 3-424 (WES 1956b). It was also noted by Daniel (1985) in his analysis of Rock Island performance data. In fact, there is a significant similarity between the TM 3-424 figure and Daniel's figure. Nevertheless, boil occurrence is rare in terms of the many miles of levee subjected to similar gradients. It is apparent that local geologic conditions must have a more significant influence on where boils occur than does the gradient. There is considerable evidence that boil occurrence is often related to concentration of seepage at discontinuities and defects in the top blanket. Such nonuniform blanket geometry is not accounted for in the uniform, 2-D model used for design. Despite the verbiage given to geologic conditions in TM 3-424 and the colorful 3-D cross sections illustrating floodplain deposits and their relationship to underseepage, the same analysis and design criteria are applied in the same manner for all types of deposits. The likelihood of boil occurrence at discontinuities is also implied by Cunney (1980) who refers to a long-held concern that berm formulas... are not appropriate for locations where seepage pressures can be uniformly and harmlessly dissipated (emphasis added).

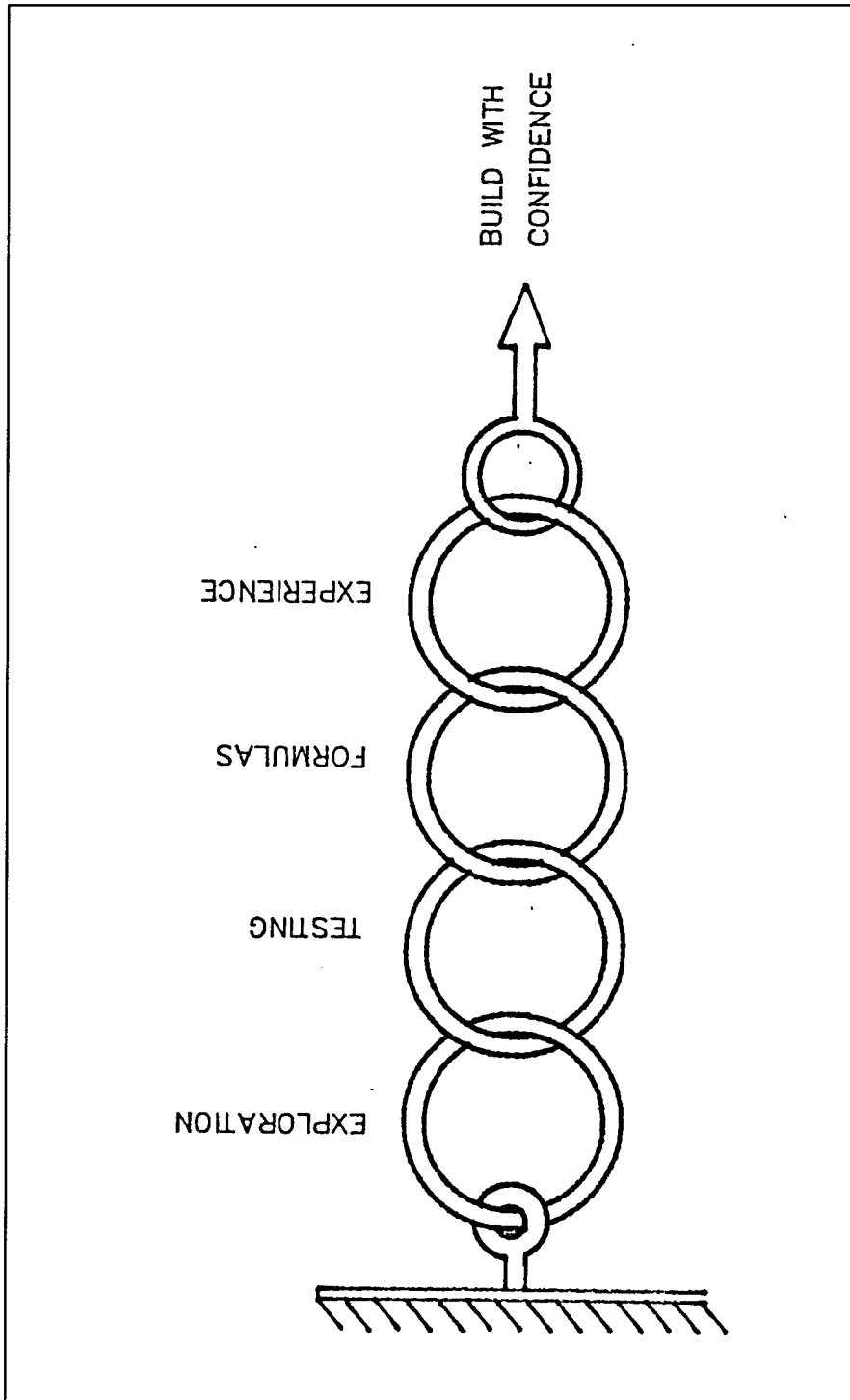


Figure 2. The design chain (after Harr 1977)

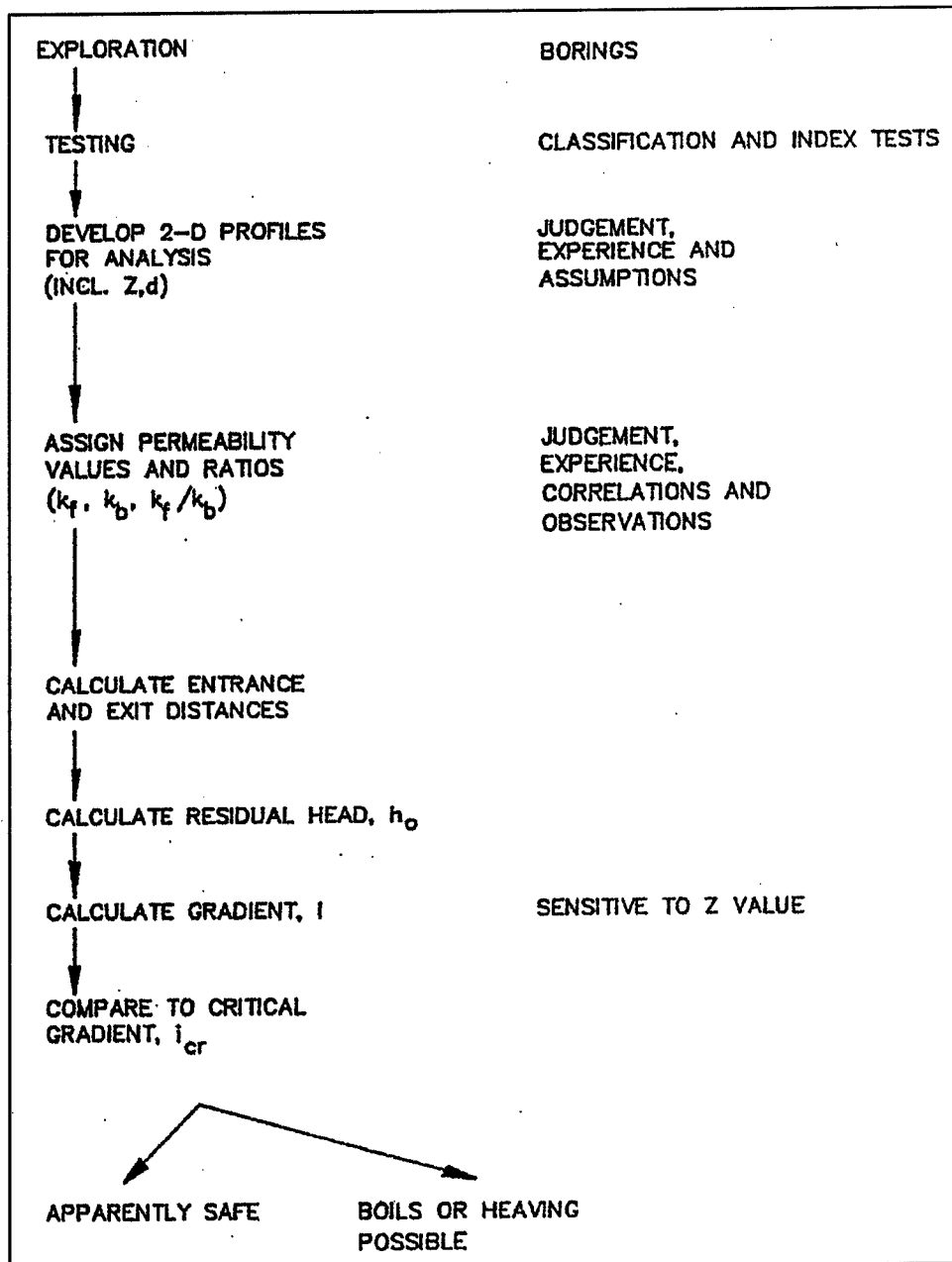


Figure 3. The analysis chain for underseepage

Relationship of Boils to Blanket Thickness

The correlation presented by Daniel (1985) between boil occurrence and top blanket thickness implies that boils are the only concern and overlooks the possibility of rather sudden rupture of thick clay blankets retaining high piezometric pressures (heaving). This was apparently the case of the 1943 floodwall failure at Claryville, MO, described by Middlebrooks and Jervis (1947). Safety, seepage quantities, and pressures are related to both blanket thickness and blanket permeability. These relationships are conceptually illustrated in Figure 4.

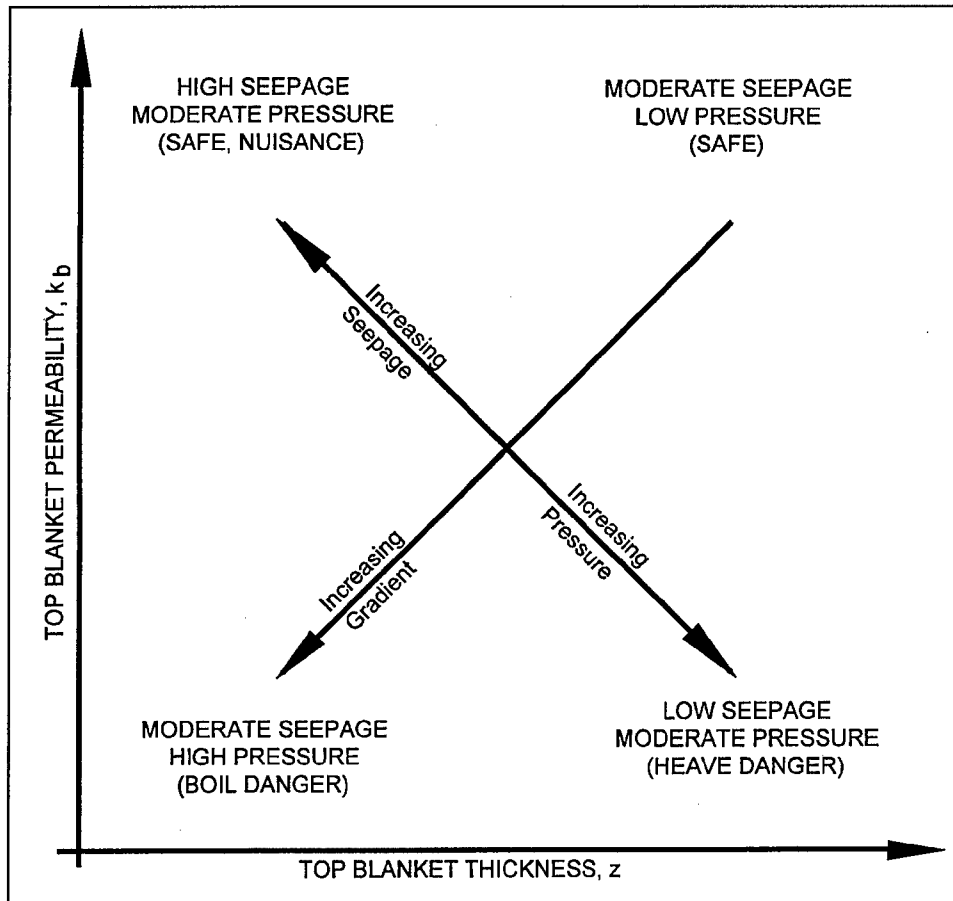


Figure 4. Relationships of seepage and subsurface pressures to blanket properties

Critical Gradient Criteria

Daniel (1985) notes that the calculation of the critical gradient was based on a homogenous top blanket with no cohesion and flexural strength and noted that these assumptions would often be invalid. This was also challenged in a discussion of Turnbull and Mansur's paper (1961b). In this discussion was recommended the use of a factor of safety against uplift defined as the ratio of

the saturated weight of the blanket to the piezometric pressure at the base of the blanket. In their reply, Turnbull and Mansur did not argue the concept where the gradient safety factor is greater than the uplift safety factor, but pointed out that it is more conservative to use the gradient safety factor where it is the lower value. Furthermore, they conceded in the case of cohesive clay blankets, particularly in ditch bottoms in which the span is relatively short, the blanket might be sufficiently tough and cohesive to hold a pressure somewhat greater than the critical. However, it does not appear prudent to rely on cohesive strength in most cases for design practice.

Calculation of Gradients

As pointed out by Daniel (1985), the calculation of gradients is an uncertain process because of the difficulty in properly estimating the blanket thickness, z . It becomes very judgmental where a nonhomogeneous blanket must be transformed to an equivalent homogeneous blanket, or where the blanket changes thickness along or beyond the levee toe. In ridge and swale topography, the top blanket may be highly stratified, and development of an idealized design profile by the engineer may seem to be a meaningless process. Estimating blanket thicknesses was a constant problem for the USAED, St. Louis (1976) analysis. Local geology enters the picture again; the equations can predict adverse seepage conditions only to the extent that the section analyzed models the subsurface conditions.

Calculation of Entrance and Exit Distances and Residual Head

Daniel's review suggested that accurate values of the entrance distance, s , and the exit distance, x_3 , are almost impossible to obtain. The problems are not as severe in practice as it would appear, even though they are functions of four uncertain parameters. This arises because the prediction of interest is the residual head, h_o , at the levee toe. Working backwards through the analysis equations, h_o is determined by simple proportion involving the entrance and exit distances:

$$h_o = H \frac{x_3}{x_1 + x_2 + x_3} \quad (1)$$

where

x_2 = base width of the levee

$x_1 + x_2$ = entrance distance, s , from the landside toe

It is apparent that h_o can be accurately calculated if the proportion between x_1 and x_3 is reasonably correct, even if their actual values are grossly in error. For a levee reasonably distant from the river,

$$x_{1,3} = \sqrt{\frac{k_f}{k_b}} zd \quad (2)$$

where riverside values of the parameters are used to calculate x_1 and landside values are used to calculate x_3 .

As the landside and riverside values are often significantly correlated, the equations yield values for the entrance and exit distances that are generally in correct proportion. Furthermore, the extraction of the square root tends to minimize the effects of error in the parameters, and errors in z and d are just as likely to be compensating as biased. Cunny (1980) implies the same idea; that is, that one can reasonably predict the residual head even with the wrong permeability ratios.

The variation of x_1 and x_3 with river stage noted by Daniel (1985) is discussed in detail in TM 3-424, Appendix E (WES 1956b). Although the analysis procedure requires a constant value, it is inferred that the design value should represent the critical combination of x_1 and x_3 values.

Permeability Values and Ratios

Although hydraulic conductivity (or permeability) is difficult to quantify, the Corps' recommendations are not arbitrary as suggested by Daniel (1985) but are based on considerable experience and piezometric measurements. Residual heads and gradients are dependent only on the ratios of the permeabilities, not their absolute values. As the values used are back-calculated from observed piezometric grade lines and then reused in the same equations to estimate the piezometric grade line for other conditions, it is not surprising that they provide generally good results. The permeability ratios and the blanket formulas form a closed-loop; thus, they tend to work whether they are correct or not.

Nevertheless, data obtained from the 1973 flood in St. Louis indicated lower ratios than those typically recommended for use in the Lower Mississippi Valley, and the Rock Island analysis indicated still lower values. While the reasons for this trend require more study, it is noted that these sites represent significant differences in the geologic environment. The Lower Mississippi is a classic meandering stream in a wide valley. Levees are at relatively great distances from the river, and discontinuities such as clay plugs and oxbows are common. The river carries a high sediment load. At the other extreme, the characteristics of the valley in the Rock Island District are primarily related to glacial melting. The valley is rather narrow and there are relatively few meander deposits. Levees are relatively close to the river. Much of the sediment load enters the river downstream of the Rock Island District. The St. Louis District and the middle Mississippi Valley represent transitional conditions. Concentrations of seepage adjacent to clay plugs or other blanket discontinuities increase residual heads and may result in apparently higher permeability ratios than would be measured under relatively uniform blanket conditions.

Determination of Parameters from Piezometer Data

Estimates of entrance distances, exist distances, and permeability ratios have generally been made only at piezometer ranges because a linear hydraulic grade line can be fitted through a number of points. Too many assumptions appear necessary to estimate these factors from a single piezometer at the levee toe. Nevertheless, all reports of such analyses have mentioned the difficulty in obtaining reasonable values because of the sensitivity of the calculations to minor errors in the differences between piezometer readings. In an effort to assess the relative importance of the variables used in the analysis, a simplified form of the equations yielding the landside residual head was developed (Appendix A). Using this equation and the measured residual head from a single piezometer at the levee toe, and making a few reasonable assumptions, considerable insight can be gained regarding the probable values of x_1 , x_3 , and the permeability ratio. This item is further discussed in Appendix A.

Assumptions of Vertical and Horizontal Flow

Daniel (1985) points out the weakness of the assumptions of a constant head along vertical planes in the pervious substratum (horizontal seepage). The validity of this assumption increases with increasing permeability ratio; Bennett (1946) warned of the necessary conditions for making this assumption. The error resulting from this assumption was investigated in TM 3-424 (WES 1956b), and data were presented to show that there is generally less than a 0.61-m (2-ft) head difference between piezometers at the base of the blanket and at the midpoint of the aquifer. However, the problems associated with silty sands in an intermediate aquifer noted in the St. Louis analysis and similar problems expressed by U.S. Army Engineer District, New Orleans (personal communication), in silty sands support Daniel's concern.

Deficiencies in Procedures, Summary

Based on the various reviews of performance data, a summary of the assumptions made in underseepage analysis and the special cases in which they may be deficient has been prepared and is given in Table 1. The performance data also indicate that there can be wide variation in the observed values of parameters assumed or calculated in the design. To illustrate this, the ranges of the permeability rates and entrance distances are shown in Figure 5.

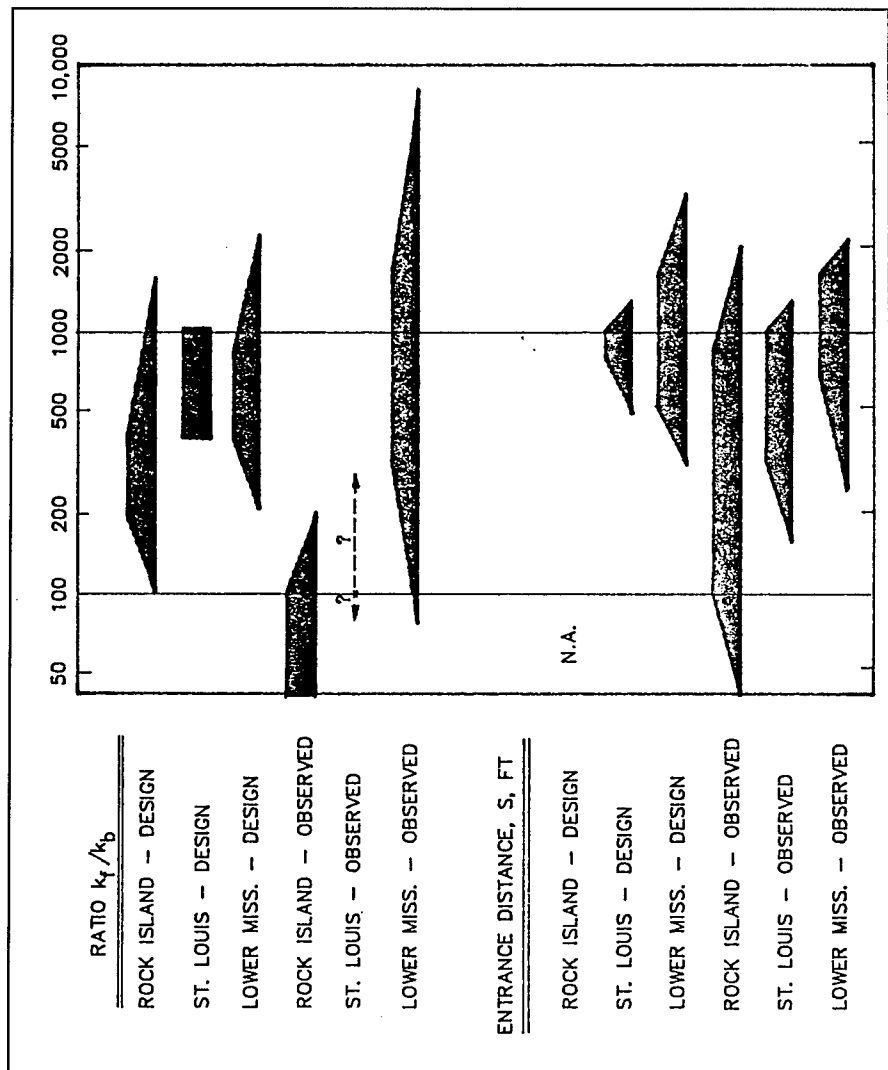


Figure 5. Typical ranges of parameters (k_f/k_b) and s

Possible Improvements to Procedures

Possible improvements to the analysis procedures lie in four areas:

- a.* Computerized analysis using existing procedures to allow more expedient solutions.
- b.* Probabilistic adaptation of existing procedures to allow for uncertainty in the parameters.
- c.* Extension of the existing procedures to more general cases to allow more realistic modeling of actual conditions.
- d.* Improvements in the exploration process to allow better identifications of the subsurface conditions to be modeled.

The equations for seepage analysis as well as for design of seepage berms and relief wells have been adapted to computer programs by several parties, including Mr. Patrick Conroy of the St. Louis District and one implemented by Jaycor, Inc., for the ERDC. Daniel's (1985) recommendation of a relatively sophisticated computer program is vague. As he notes what he believes are significant deficiencies in the present analytical technique, it is not what should be computerized.

Barron (1984) suggested that the uncertainty of the variables was the problem rather than the analytical techniques and suggested development of a probabilistic approach.

However, Table 1 has identified several cases where the uniform, 2-D idealized profile used in analysis is incompatible with the actual subsurface conditions. Suggested areas of improvement in this regard (items *c* and *d* above) are noted in Chapter 9 of this report.

7 Problems with Underseepage Monitoring and Controls

Maintenance of Piezometers

In all regions studied, there have been numerous occasions of piezometers being damaged by tractors and other vehicles, and piezometers malfunctioning because of siltation and blockage by foreign substances. All of the major performance review reports cite problems with faulty piezometers. A rotating maintenance program is employed by the St. Louis District that provides for inspection and repair of every piezometer over a 3- to 5-year cycle.

Piezometer Reading During Flood

Review of the various performance reports indicates that methods for determining which piezometers to read and when to read them varies considerably from District to District and from time to time. Several reports recommend daily piezometer readings during flood. Such frequency was nearly accomplished by the St. Louis District during the 1973 flood; however, emphasis on quantity of readings can cause engineers to be inundated with data with the attendant risk of a failure occurring while the data are waiting to be analyzed.

Piezometer reading during flood has two somewhat contrary objectives real-time safety assessment and the evaluation of design procedures using performance data. In the first case, the emphasis should be on wide-scale coverage and visual assessment of the levee system by trained geotechnical engineers. In this case, the engineers read piezometers only to the extent necessary to assess safety, and piezometers are selected based on such concerns as high apparent pressures (flowing piezometers), known problem areas based on previous performance, and areas with new levees and no previous experience. In the second case, procedure assessment, the emphasis should be on obtaining complete information, but only in carefully selected areas where the subsurface conditions are sufficiently well-defined to permit detailed analysis. Experienced technicians may be used for acquiring data as the analysis may be done at a later date. Equally important to both cases is the measurement of river levels and landside impoundment levels as

well as piezometric levels; these measurements have been often cited as being overlooked in the performance review reports.

Premature Relief Well Flow

A significant operational problem with relief wells has been that they begin to flow at overbank, but noncritical, river stages. Where collection and disposal measures are not provided for well effluent, such as along certain agricultural levees in the St. Louis District, crop damage may occur during normal spring high water. Consequently, farmers have obstructed well outlets with lumber, sandbags, and other devices, posing a potential threat to underseepage control. Beginning with the record 1973 flood, local interests have been cooperative in removing obstructions as significant river stages occur and when advised by field engineers. The only solution employed to date has consisted of providing the wells with a removable plastic standpipe that prevents premature flow but reduces the factor of safety. There has been considerable discussion over the years, but little research and development, on providing a positive but foolproof valving system that would open when needed.

Deterioration of Relief Wells

Historically, there has been concern with the use of relief wells for underseepage control because of possible reduction of efficiency over time resulting from screen incrustation. A detailed discussion of the problem is beyond the scope of this report. To evaluate possible reduction in efficiency, pumping tests have been periodically performed on wells in the Lower Mississippi Valley (WES 1952) and in the St. Louis District (Montgomery 1972; USAED, St. Louis 1976). The earlier reports document a reduction in well efficiency (sometimes substantial) with time but at a decreasing rate. The later report and subsequent unpublished studies in the St. Louis District indicate that prolonged well flow and changes in groundwater chemistry during flood may lead to recovery of lost efficiency at the time the wells are most needed.

8 European Practice

Peter (1982) provides a thorough review of underseepage analysis techniques employed in Europe, with particular emphasis on the Danube Valley. The differential equations for landward propagation of seepage pressures through layered anisotropic foundation soils are presented and numerical solution is suggested. Also, a discussion is presented regarding the prediction of sandboil occurrence using the critical gradient versus the critical velocity approach. In the critical velocity approach, soil properties (grain and/or pore diametric porosity, grain size distribution) and water velocity are considered in addition to unit weight.

It is apparent that European engineers have continued theoretical and laboratory research beyond the 1950's methods used by the USACE. However, the available presentations are highly theoretical and not amenable to practice in their present form; also the fourth link (experience) of the design chain is not present for American levees and soils. Any new research and development in underseepage analysis and control should include a careful review of European research and practice.

9 Conclusions and Recommendations

Conclusions

To prevent sandboils and heaving of the top blanket, the USACE has devised a system of underseepage analysis procedures and control measure design procedures. The analysis procedures seek to find reaches where the exit gradient at design flood would exceed a critical value (typically 0.85),¹ based on foundation properties and geometry and the assumptions described in Table 1. In the reaches found, the USACE designs and constructs control measures (seepage berms or relief wells) that are costly and have associated operational problems.

Based on data from St. Louis, Rock Island, and the Lower Mississippi, it can be fairly stated that boils occur in locations that are primarily governed by minor geologic details and discontinuities. Where they do occur, they are associated with apparent gradients of 0.5 to 0.9, often on the order of 0.7.

Although the local geology is identified as being of great importance in the development of underseepage problems, in practice it is incorporated into the analysis procedure only in a very indirect and judgmental manner and may often be overshadowed by the number-crunching aspects of the design. The uniform, 2-D cross section used in analysis is incapable of predicting seepage conditions in nonuniform or discontinuous profiles.

In the writer's opinion, the present procedures in practice probably identify most of the reaches where underseepage may be critical and probably misidentify many more reaches. On the other hand, they probably miss a few critical reaches which then require remedial treatment during flood. To hazard an educated guess, the reader is referred to Table 2, which is based entirely on the writer's experience and opinion, and is intended to illustrate defects in the analysis chain more than to present defensible numbers.

A critical weak point in the entire analysis and design process is the characterization of the top blanket. No reasonable and consistent method is available that will lead two designers with the same boring log to necessarily

¹ Districts have lowered the critical exit gradient to 0.5 since the great flood of 1993.

Table 2 Estimated Adequacy of Seepage Analysis Procedures¹			
Identified	Actually Critical	Actually Noncritical	Total
Critical	19%	21%	40%
Noncritical	1%	59%	60%
Total	20%	80%	100%
¹ Figures are estimated percentages of levee length.			

similar values of z and k_b , yet the calculation process is driven by these variables. The blanket profile is often developed by borings 152 m (500 ft) apart, sampled on 0.9- to 1.5-m (3- to 5-ft) increments. Continuity of lenses and layers in the top blanket is usually uncertain. Division of the levee profile into design reaches is an undocumented “art.” Much of the success of present designs might be attributable to the fact that much of the design was accomplished by a relatively small group of engineers also involved in the development of the equations and criteria.

Parameters such as the permeability ratio between the foundation and the top blanket seem to exhibit significant variations going from the Upper Mississippi River to the Lower Mississippi River and there is reason to hypothesize that such differences result from the depositional environment of the materials. More detailed research in this regard may yield a more rational approach to estimating such parameters.

Development of supplemental analytical techniques would be useful for certain situations listed in Table 1. If the present procedures are to be revised with a view toward reduction of the number of reaches requiring controls, the emphasis should be on the geometry, characteristics, and continuity of the top blanket.

Recommendations¹

To update underseepage analysis and control techniques for their second 50 years, research recommendations are offered in the three areas of analysis, design and construction, and expedient control during floods.

Analysis

Apparent research needs include the following:

- a. Development of a 2-D analysis procedure incorporating three foundation layers, each with anisotropic permeability conditions (Figure 6).

¹ Some recommendations are out of date, because this report was written in 1989 and published 2002.

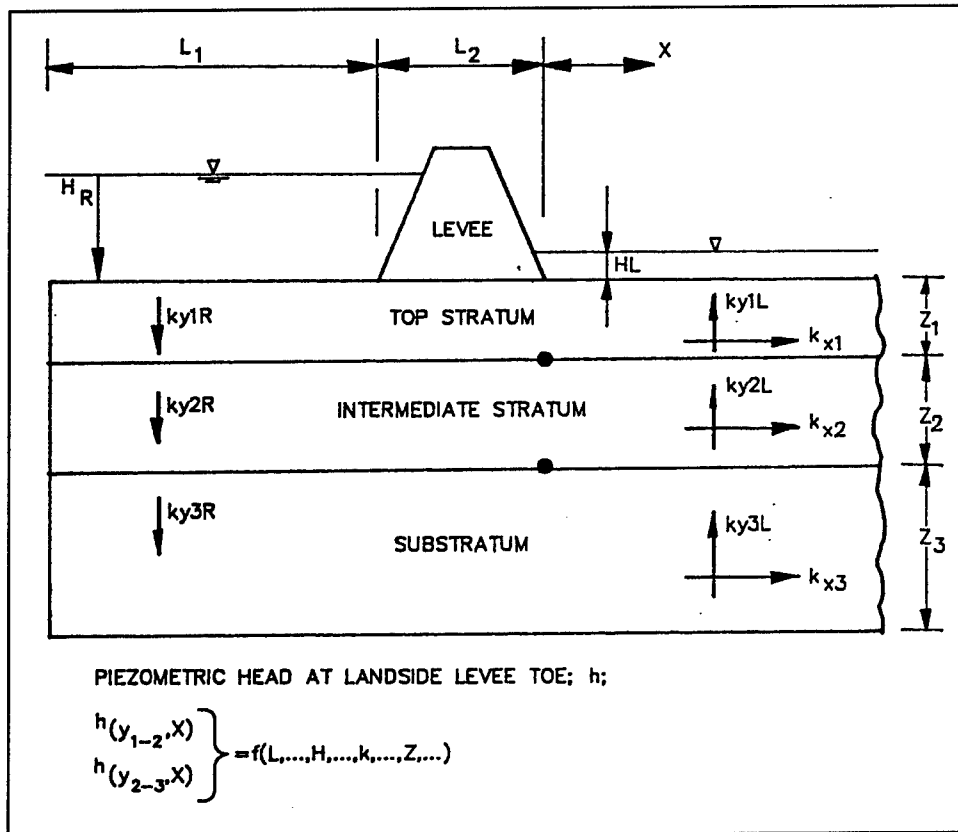


Figure 6. Analysis model for three-layer foundation

- b. Development of a 2-D analysis procedure for nonuniform foundation conditions such as borrow pits, ditches, and clay plugs parallel to the levee (Figure 7).
- c. Development of an analysis procedure for levee corners (Figure 8).
- d. Development of a general, 3-D analysis procedure (Figure 9).
- e. Development of an analysis procedure accounting for time effects.
- f. Development of probabilistic analysis procedures that consider uncertainty in the variables.
- g. Research into better techniques to characterize the top blanket and subdivide reaches. The cone penetrometer and shallow geophysical techniques offer the capability to significantly increase the level of information normally obtained by conventional borings.

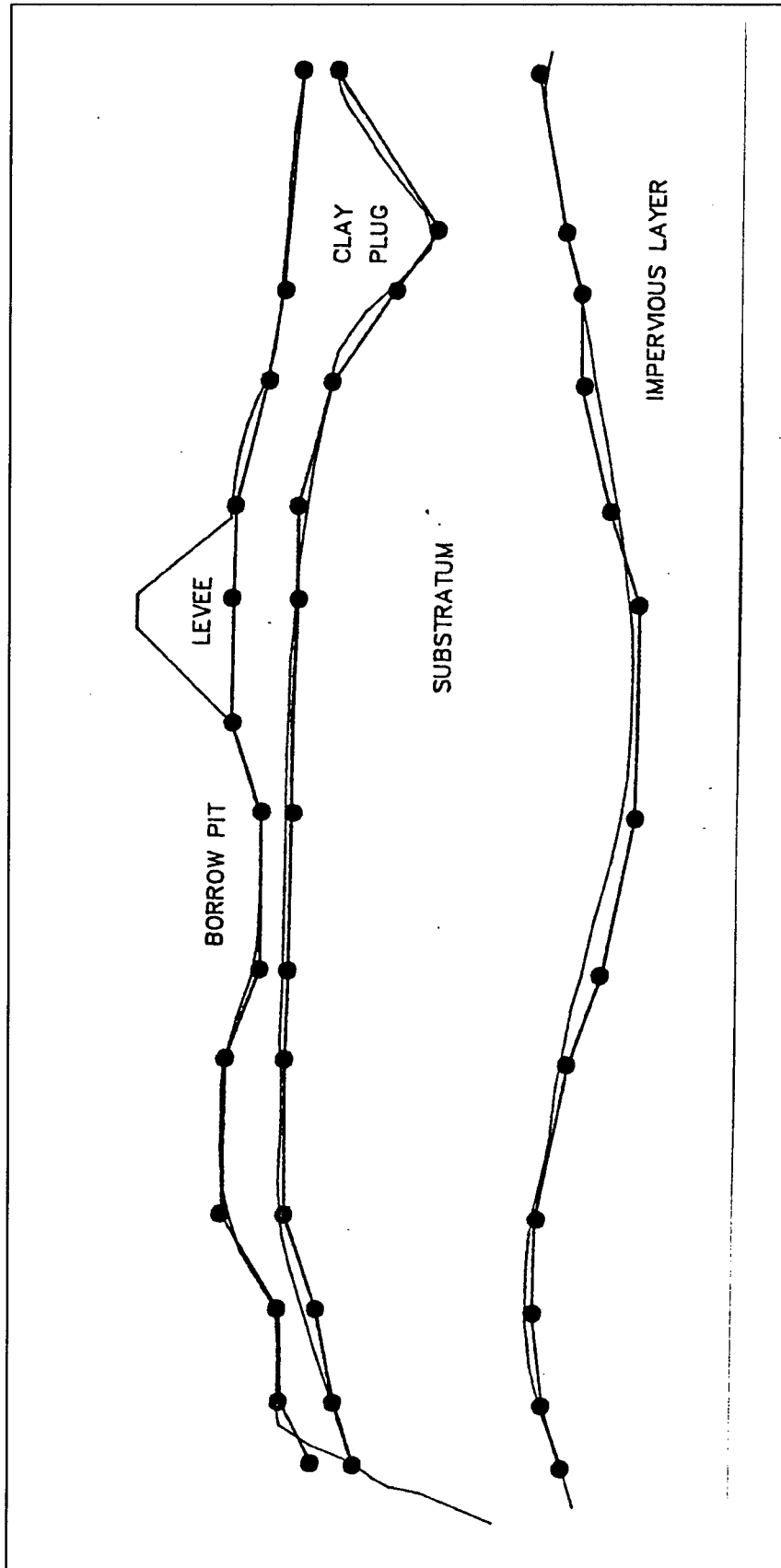


Figure 7. Analysis model for nonuniform foundation materials

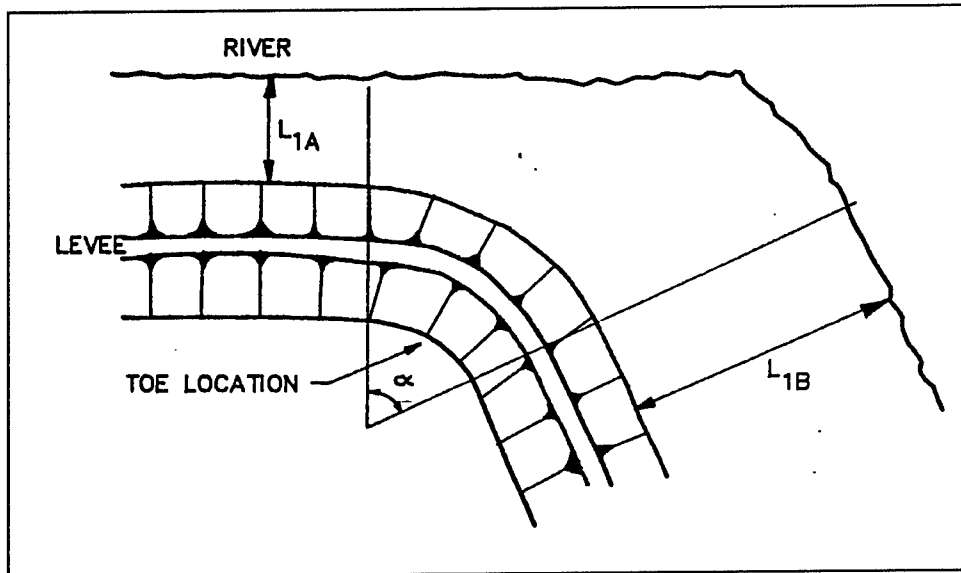


Figure 8. Analysis model for levee corners

Design and construction

Apparent research activities include the following:

- a. Entrance losses for modern well screens. Designers still use curves for wooden well screens with a 572-sq cm per meter (30-sq in.) open area per foot. Modern wire-wound well screens provide open areas in excess of 1,908 sq cm per meter (100 sq in. per foot). This higher efficiency is not incorporated in design because reliable head loss data are not available.
- b. Design and construction of shallow jetted wells. It may be cost-effective to construct lines of partially penetrating wells installed by jetting similar to the techniques used for installing suction wells for dewatering. Although more wells would be required, the savings in drilling and filter placement may likely result in a net savings.
- c. Use of continuous prefabricated vertical drains (similar to prefabricated wall drains) along the levee toe.

Expedient control during floods

Traditional sand bag ringing of sandboils is labor-intensive, time-consuming, and hazardous to personnel. Many other techniques for boil control could be conceived and evaluated. These might include:

- a. Weighted geotextile blankets.

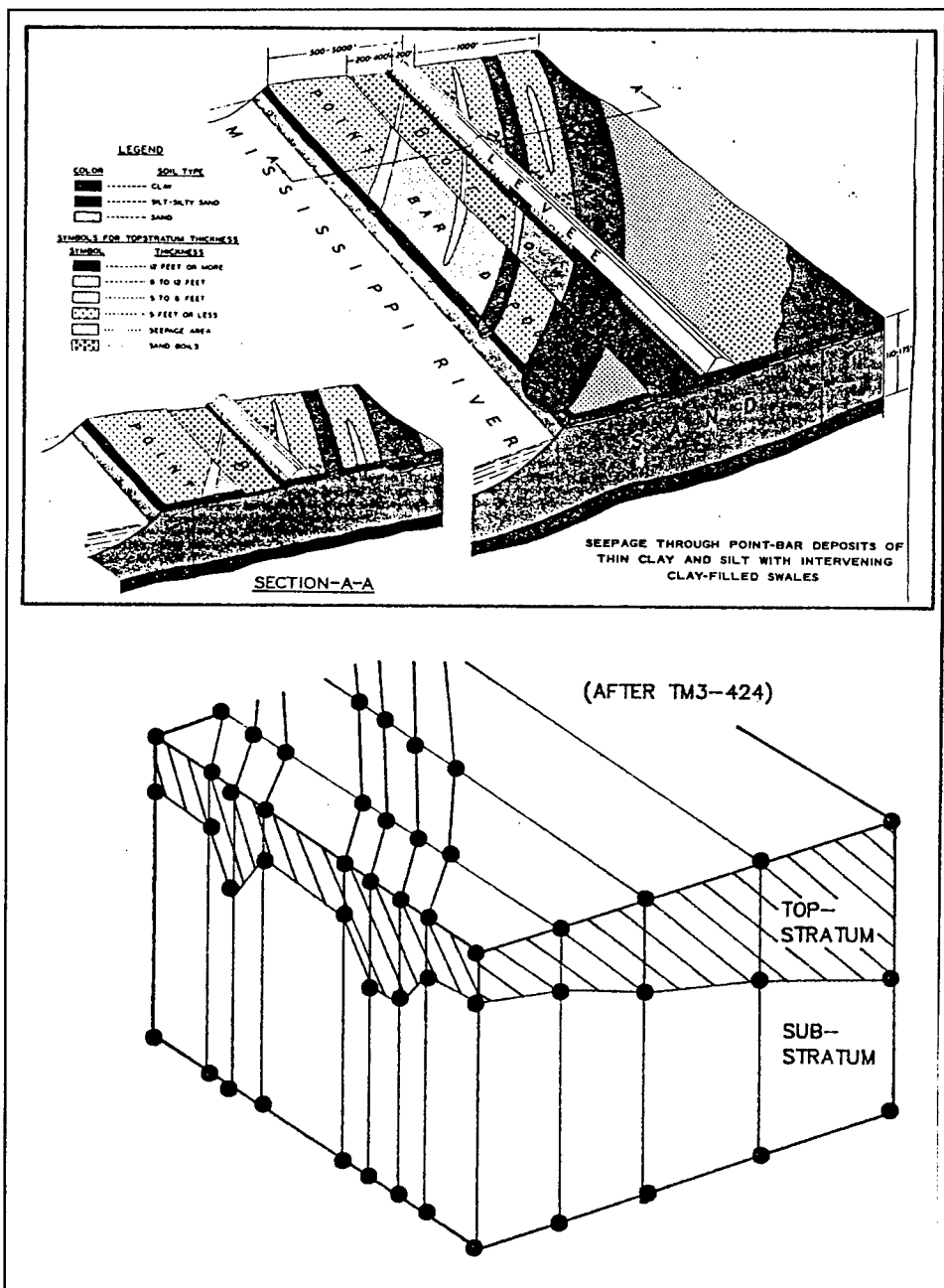


Figure 9. Analysis model for 3-D geometry

- b. Dropping or driving a well screen into the boil.
- c. Additional perforation of the top blanket.

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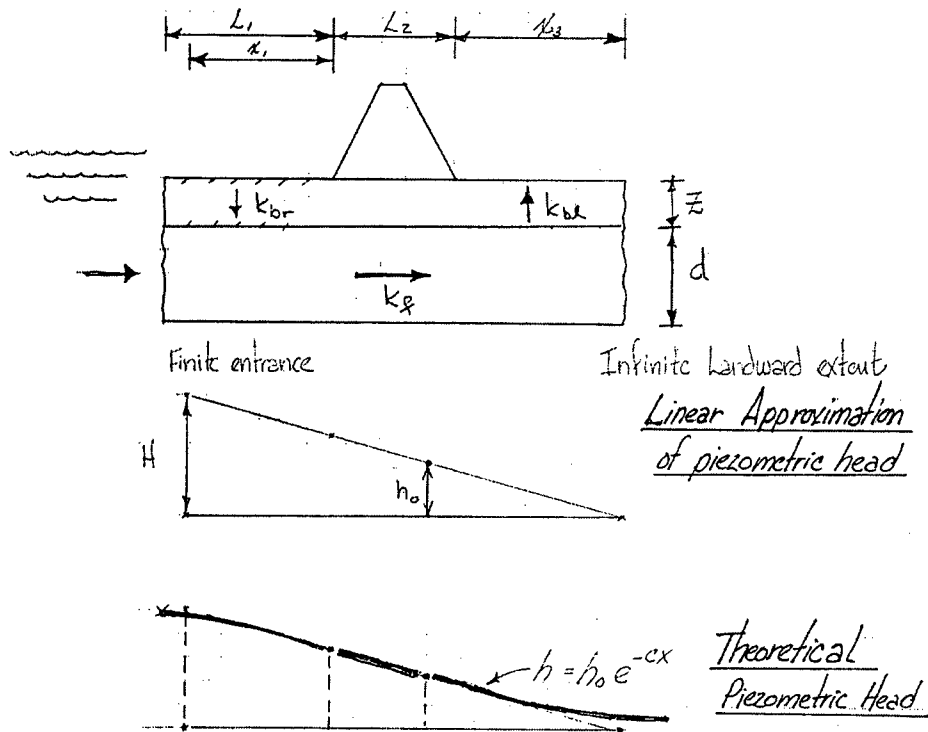
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Appendix A

Derivation of Alternate Equation for Residual Head

EQUATION FOR HEAD AT LANDSIDE TOE



EQUATION FOR h_0 :

$$\left(\frac{h_0}{H}\right) = [\tanh(\beta C_t L_1) + C_t L_2 + 1]^{-1}$$

where $C_t = \sqrt{\frac{k_{br}}{k_f \cdot z \cdot d}}$ a "resistance" factor

and $B = \sqrt{\left(\frac{k_{br}}{k_{be}}\right)\left(\frac{z_t}{z_r}\right)} \approx \sqrt{\left(\frac{k_{br}}{k_{be}}\right)}$ a "blanket factor"

DERIVATION OF RESIDUAL HEAD EQUATION

$$h_o = \frac{H x_3}{S + x_3}$$

$$\left(\frac{h_o}{H} \right) = \left(\frac{x_3}{S + x_3} \right) = \frac{x_3}{x_1 + L_2 + x_3}$$

$$C_\ell = \left(\frac{1}{x_3} \right) = \sqrt{\frac{k_{bl}}{k_f \cdot z_{tl} \cdot d}} \quad C_r = \sqrt{\frac{k_{br}}{k_f \cdot z_{tr} \cdot d}}$$

$$\frac{C_r}{C_\ell} = \left(\frac{k_{br}}{k_f \cdot z_{tr} \cdot d} \right)^{1/2} \left(\frac{k_{bl}}{k_f \cdot z_{tl} \cdot d} \right)^{-1/2}$$

$$\frac{C_r}{C_\ell} = \sqrt{\left(\frac{k_{br}}{k_{bl}} \right) \left(\frac{z_{tl}}{z_{tr}} \right)} = \beta, \text{ the BLANKET FACTOR}$$

$$\therefore C_r = \beta C_\ell$$

$$\left(\frac{h_o}{H} \right) = \frac{x_3}{x_1 + L_2 + x_3}$$

$$\left(\frac{h_o}{H} \right) = (x_3)(x_1 + L_2 + x_3)^{-1}$$

$$\left(\frac{h_o}{H} \right) = \left(\frac{1}{C_\ell} \right) \left(\frac{\tanh(C_r L_1)}{C_r} + L_2 + \frac{1}{C_\ell} \right)^{-1}$$

$$\left(\frac{h_o}{H} \right) = \left(\frac{1}{C_\ell} \right) \left(\frac{\tanh(\beta C_\ell L_1)}{C_\ell} + \frac{C_\ell L_2}{C_\ell} + \frac{1}{C_\ell} \right)^{-1}$$

$$\left(\frac{h_o}{H} \right) = [\tanh(\beta C_\ell L_1) + C_\ell L_2 + 1]^{-1}$$

Example to assess proper design parameters.

Measure $\left(\frac{h_o}{H}\right), L_1, L_2$

Find (β, C_t values that satisfy solution, use $\beta = 0.1, 0.2, 0.4, 0.7, 1.0$)

Then, knowing d and z , estimate

$$\left(\frac{k_f}{k_{bt}}\right), \left(\frac{k_{br}}{k_{bt}}\right) \text{ for } z_{it} = z_{cr}, \beta = \sqrt{\frac{k_{br}}{k_{bt}}}$$

Example: Rock Island, Sky Island, Range B:

Piezometer B3,

$$\text{On May '65, } \frac{h_o}{H} = \frac{2.7}{466.8 - 458.2} = 0.314$$

$$L_1 \approx 200', L_2 \approx 200'$$

$$0.314 = [\tanh(\beta C_t 200) + C_t(200) + 1]^{-1}$$

$$\tanh(200 \beta C_t) + 200 C_t + 1 = 3.1852$$

$$\tanh(200 \beta C_t) + 200 C_t = 2.1852$$

Assume $\beta = 0.1$ ($k_{bt} = 100 k_{br}$)

$$\tanh(20 C_t) + 200 C_t = 2.1852$$

Solve by iteration:

Try $C_t = 0.0010$

$$\tanh(0.02) + (200)(0.001) = 0.0200 + 0.200 = 0.2200$$

Try $C_t = 0.0100$

$$\tanh(0.2) + 2 = (0.1974) + (2) = 2.1974$$

Try

$$C_t = 0.0090$$

$$\tanh(0.18) + 1.80 = 0.17808 + 1.800 = 1.9781$$

Try $C_t = 0.0099$

$$\tanh (0.198) + 1.98 = 0.1964 + 1.98 = 2.1764$$

Use $C_t = 0.0099$ for $\beta = 0.1$

$$0.0099 = \sqrt{\frac{1}{\left(\frac{k_f}{k_o}\right) \cdot z \cdot d}} \quad z = 7.4', \quad d = 34'$$

$$\left(\frac{k_f}{k_b}\right) = 41$$

Assume $\beta = 0.7071$ ($k_{bl} = 2 k_{br}$)

$$\tanh ((200)(0.7071) C_t) + 200 C_t + 1 = 3.1852$$

$$\tanh (141.4 C_t) + 200 C_t = 2.1852$$

Try $C_t = 0.0099$

$$\tanh (1.3999) + 1.98 = 0.8853 + 1.9800 = 2.8653$$

Try $C_t = 0.0090$

$$\tanh (141.4 \times 0.009) + (200)(0.009) =$$

$$\tanh (1.2726) + 1.8000$$

$$0.854 + 1.8000 = 2.654$$

Try $C_t = 0.0050$

$$\tanh (141.4 \times 0.005) + (200)(0.005) =$$

$$0.608 + 1.000 = 1.608$$

Try $C_t = 0.0070$

$$\tanh (141.4 \times 0.007) + (200)(0.007) =$$

$$0.756 + 1.400 = 2.156 *$$

Try $C_t = 0.0065$

$$\tanh (141.4 \times 0.0065) + (200)(0.0065) =$$

$$0.720 + 1.300 = 2.0200$$

Try $C_t = 0.0067$

$$\tanh (141.4 \times 0.0067) + (200)(0.0067) =$$

$$0.739 + 1.34 = 2.079$$

$$0.0070 = \sqrt{\frac{1}{\left(\frac{k_f}{k_b}\right) \cdot 7.4 \cdot 34}}$$

$$\left(\frac{k_f}{k_b}\right) = 81$$

For the range of assumptions on β , we find:

$$k_{bt} = 100 k_{br} \quad \beta = 0.1 \quad \left(\frac{k_f}{k_{bt}}\right) = 41$$

$$k_{bt} = 2 k_{br} \quad \beta = 0.707 \quad \left(\frac{k_f}{k_{bt}}\right) = 81$$

$$\text{Cunny (1980)}^1 \text{ (p. 101) finds } \left(\frac{k_f}{k_{bt}}\right) = 54$$

* The ratio $\left(\frac{k_f}{k_{bt}}\right)$ can be bounded knowing $L_1, L_2, z_{bt}, d, \left(\frac{h_o}{H}\right)$

* Need data from one piezometer to get $\left(\frac{h_o}{H}\right)$

* Assume $0.1 < \beta < 0.707$

¹ References are listed on page 32 of main text of this report.

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14. ABSTRACT The Federal Government through the U.S. Army Corps of Engineers (USACE) has a large investment in flood-control levees. Where such levees are built on pervious foundations, seepage beneath the levee (underseepage) during floods can produce pressure and flow conditions capable of initiating subsurface erosion leading to levee failure. Two adverse phenomena may occur; one is sand boils which involves the movement of subsurface sand to the surface by flowing water, and the other is heaving which involves the upward movement of a relatively impervious surface layer resulting from subsurface water pressures in excess of its weight. To prevent such occurrences, the USACE has developed a set of procedures to analyze underseepage conditions on a site-specific basis and a set of procedures to design underseepage control measures. For the most part, these procedures were developed in the 1940s and 1950s. Intensive construction of control measures was accomplished in the 1950s and 1960s. Several moderately large and major floods have provided data from which the validity of the procedures and the security of the constructed system can be inferred. Also, since the 1950s many technical advancements have been made in engineering analysis techniques and construction methods that may merit application to underseepage problems.					
<i>(Continued)</i>					
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The Federal Government's levee system will be expected to provide flood protection for many centuries, regardless of its so-called economic life. It will undoubtedly be subjected to floods equaling and exceeding those already experienced. Conditions along the levees are not static but are subject to periodic natural and man-made changes. Such changes may necessitate review, reanalysis, redesign, reconstruction, and/or modification of the system.

Several researchers have prepared voluminous evaluations of the performance of particular levees in particular floods. This report draws on those previous assessments to summarize in one source what has been learned from observations during floods up to 1986. Using that knowledge, the analysis procedures and the performance evaluation procedures are reviewed to identify possible areas of improvement.